# BRIDGE ABUTMENTS ON ROCK

by

G.E. Rawlings* and D.C. Wyllie**

## ABSTRACT

The first section of the new Coquihalla Highway from the Fraser Valley at Hope to the B.C. Interior at Merritt traverses the rugged terrain of the North Cascade Mountains. This has required large rock excavations, the placement of substantial fills and construction of numerous bridges, culverts, retaining walls and avalanche protection measures.

This paper describes the geotechnical engineering involved with the design and construction of bridge abutments on complex rock foundations exemplified by two sites at Dry Gulch and Ottomite. Both are located on strong granodiorite and there are generally no problems of bearing capacity. However, the presence within the rock mass of numerous discontinuities (faults, joints and foliation), zones of alteration, areas of disturbance/relaxation and intrusive dykes, combine to require the careful geotechnical design of excavations for short-term and long-term stability.

A tight schedule was required on the project to meet the established deadlines. Thus the investigation of the very poorly accessible sites was limited. Overall designs were produced on the basis of geological mapping and core drilling results. However it was recognized that design modifications would be required during construction as specific geological features were uncovered.

The overall designs are described together with modifications made during construction, controls on excavations and the detailed methods of rock stabilization.

---

* Principal and ** Associate, Golder Associates, Consulting Geotechnical Engineers; Vancouver, British Columbia, Canada.

# INTRODUCTION

This paper describes the design and construction of the foundations of two bridges on the Coquihalla Highway between Hope and Merritt in south central British Columbia (Figure 1). One bridge (Dry Gulch) is a 220 m span steel arch with 70 m long approach spans and the other (Ottomite) is a three-span pre-cast concrete bridge with an overall length of 75 m. On both bridges substantial excavations had to be made to locate sound rock for the main footings and to ensure the stability of the cut slopes.

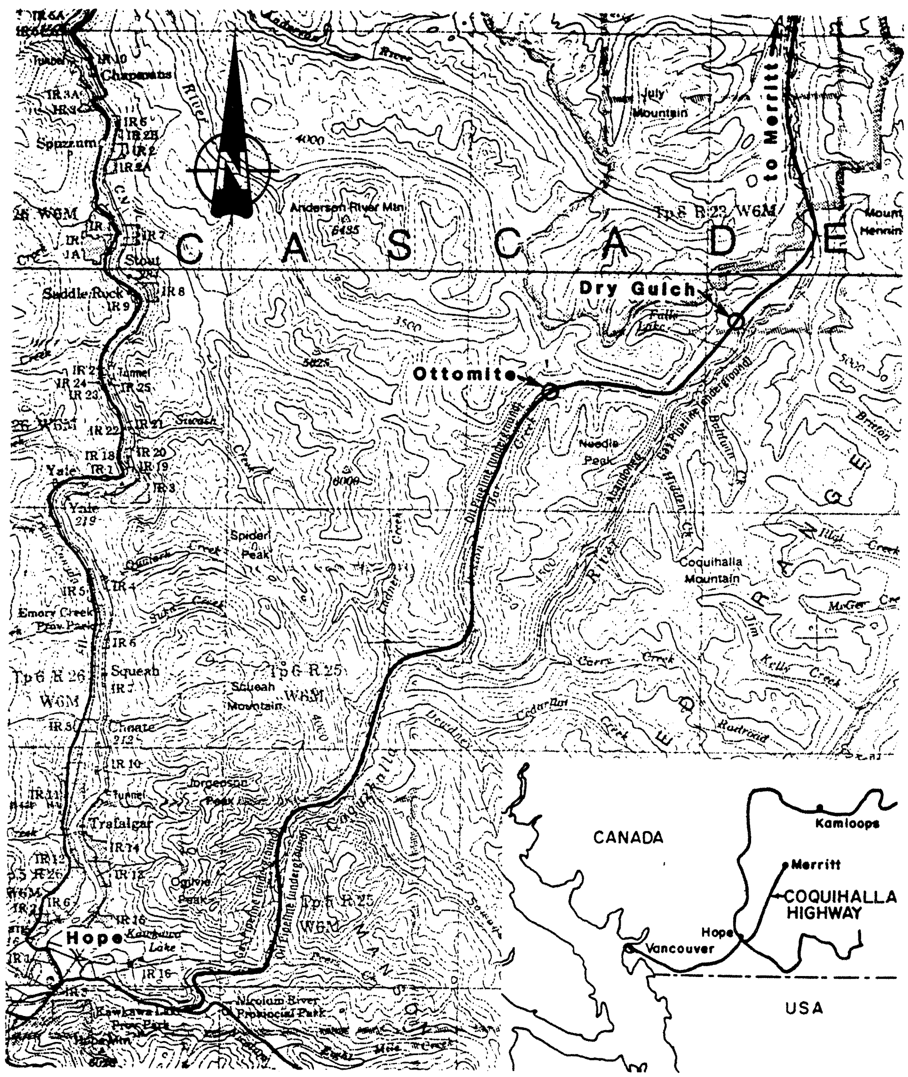



Figure 1. Site location plan.

The southern portion of this highway passes through very rugged mountainous terrain of the northern Cascades which required difficult rock excavation and the construction of many fills, retaining walls and bridges (Field et al, 1978). The difficult access to the bridge sites at the design stage, together with the tight construction schedule of one year from initial reconaissance to completion, resulted in limited time for investigation and design work. Consequently, careful examination of the excavations was required and appropriate stabilization work was carried out as the work progressed.

The predominant rock type at both bridge sites is a very strong coarse grained granodiorite, however the two sites are located in separate bodies of intrusive rock with very different geological characteristics. At Dry Gulch, the rock is part of the older and more deformed Cretaceous Eagle Granodiorite Pluton which shows a strong foliation, is faulted, closely jointed and intruded by dykes (Monger, 1967). At Ottomite, the rock is part of the Tertiary Needle Peak Pluton which is composed of massive granites and granodiorites and forms the towering peaks of the northern Cascades; joint spacing is very wide. By comparison the Eagle granodiorite at Dry Gulch produces relatively subdued topography. Despite these apparent differences in overall geology, the detailed rock mechanics problems involved in developing the bridge abutments proved to be comparable at both locations.

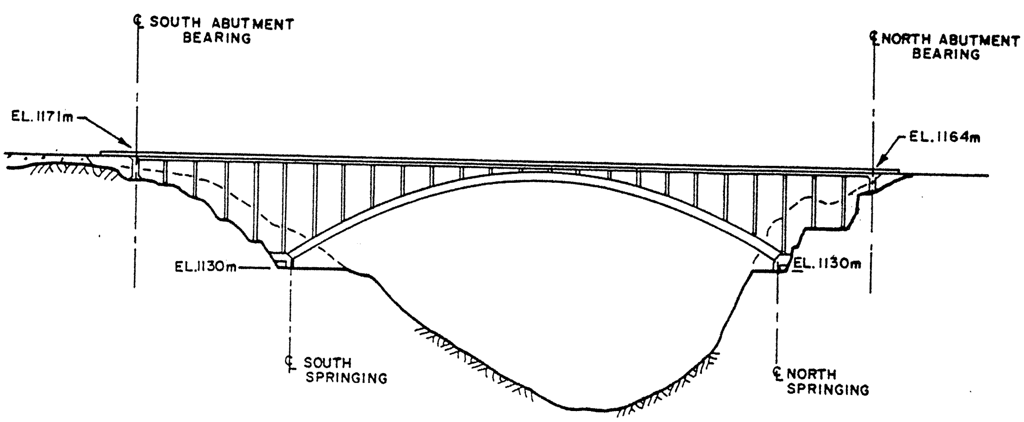
The uniaxial compressive strength of the main rock types encountered lies in the range of 150 to 250 MPa and the bearing capacity is consequently more than adequate for the loads imposed by the bridges. However, the discontinuities within the rocks at both sites dominate the geotechnical design. Despite the apparent massive nature of the rock in exposed outcrop at Ottomite, faults, joints and altered zones were found to be present in the foundations and they determined excavation levels, slope angles and the extent of slope stabilization required. At Dry Gulch the rock conditions were largely as anticipated, although specific geological features required some design modifications during construction.

## DRY GULCH BRIDGE

### PHYSIOGRAPHIC SETTING

In the Dry Gulch area the new highway is aligned north-east to southwest and runs parallel to the Coquihalla River but some 165 m higher in elevation than the river. The river has been post-glacially incised into an older broad valley creating steep and often unstable slopes. However the highway avoids this instability by keeping to the higher undulating but rugged terrain of the older valley. Dry Gulch itself is a tributary to the Coquihalla Valley and is considered to have originated as a glacial meltwater channel during the late Pleistocene. It is approximately 1.5 km long and trends towards the south-east (see Figure 2). The size of the channel completely belies the present-day catchment. A small misfit creek flows through the gulch but surface flows are only rarely seen because of the thick infilling of coarse colluvium in the bottom of the gulch. Steep rock bluffs are present along the rim on both sides of the gulch but thick colluvium marks the lower side slopes. The Dry Gulch Bridge which spans this feature is 220 m long and 90 m above the bottom of the gulch.

Looking south-east from bridge site to Coquihalla Valley.




Bridge looking upstream.

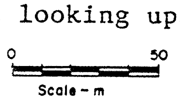



Figure 2. Dry Gulch Bridge.

## INVESTIGATIONS

Four alternative bridge designs were considered for the site: arch, rigid frame, four-span girder and cable-stayed. Each would have required different foundation support. Although access to the general area was good as construction had already been completed to the edge of the gulch from both directions, access to the valley bottom was difficult for both investigation and construction. As a result the arch alternative was selected as the preferred design. Construction of an embankment across the gulch was considered but rejected on both technical and environmental grounds.

Regional scale geological mapping was available prior to the investigations (Monger, 1967), and new exposures created by the road construction provided excellent data for the regional assessment of rock conditions. Outcrops of the Eagle Granodiorite along the road show moderately to coarse-grained, well foliated or gneissic granodiorite with common pegmatitic zones. Basic dykes, zones of intense jointing and alteration, faults and locally deep weathering are common. Glaciofluvial sands and gravels, with some tills, overlie the bedrock surface. Airphoto interpretation indicated the presence of major photolineaments, two of them intersecting at the bridge site.

Because of this geological complexity, a geotechnical investigation program was carried out with the following objectives: to confirm the proposed siting of the bridge abutments with regard to existing slope stability; to determine, as far as practicable, the foundation conditions for the arch springings, pier foundations and abutments; to collect data for the excavated slope design; and to determine the requirements for slope stabilization measures.

Geological mapping was carried out on both north and south banks using large scale oblique photographs for control. Terrestrial photogrammetry was considered and would have had technical merit, but the limited available time prior to design, precluded that option. Detailed discontinuity data was collected for slope design purposes and existing and potential slope failures were identified.

A total of 204 m of core drilling was completed in eight holes on, or close to, the springings. Drilling pads had to be blasted into the steep slopes and the rigs helicopter - lifted into place.

## GEOTECHNICAL DESIGN APPROACH

Although the bridge is founded on similar gneissic granodiorite on both the north and south sides, the degree and attitude of the discontinuities across the site is widely varied. Intrusive dykes are present to a marked extent on the north bank but are largely absent on the south. The rock material is mostly very strong and only very locally does weathering produce rock which is of sufficiently low strength to affect the design. The constraints on slope design result from the many discontinuities within the rock mass which include: foliation (uniform or highly variable with local shearing); faulting including major throughgoing faults crossing the site possibly related to the photo-lineaments, as well as minor faults within the major fault blocks), jointing (pervasive, varying from local sets through to highly persistent common sets); and diabase dykes (with sheared margins). The relationship between the orientations of the various discontinuities and the proposed excavated faces determined the slope stability.

It was concluded that in such a rock mass only the overall design parameters could be assessed from the geotechnical investigation. At best the data on which the detailed design would depend, could only be imperfectly known prior to excavation. Consequently, it was considered fundamental to the design that surveillance should be maintained during excavation to monitor departures from the assumed design conditions. This would enable the design to be modified as necessary. The slopes were to be designed to ensure long-term stability under the temporary and permanent loads imposed by the bridge.

This geotechnical design approach was considered to be particularly appropriate for the Dry Gulch situation for two reasons. Firstly, time was limited because of the tight schedule for construction. Secondly, the work was carried out on a daywork basis as an extension of the road excavation contract with the Owner taking responsibility for the construction planning. However, there were practical limitations on this approach. For example, because of pre-ordering of the steel for the arch, no design variation which would extend the length of the bridge could be tolerated.

## FOUNDATIONS

The foundation loads for design are given in Table 1.

TABLE 1 - FOUNDATION LOADS

| | Loads (kN) | Pressures (kPa) | Application |
|---|---|---|---|
| Abutment Walls | 2,224 | 385 | Vertical |
| Piers/Columns | 2,224 | 556 | Vertical |
| Arch Springing | 22,240 | 695 | 34° to the Horizontal |

Because of the need for substantial rock excavation to develop the springing foundations, the zone of weathered rock was thus removed and bearing capacity was consequently not a problem. However a deep glaciofluvial channel was intersected on the north bank and this necessitated some degree of over-excavation to achieve suitable foundations.

## SLOPE DESIGNS

The investigation of the south bank showed that there were three predominating joint sets (dip and dip direction: (1) 60/123; (2) 50/217; and (3) 70/345) which could result in potential wedge failures. A shallow zone of disturbance was mapped and indicated that a mass of rock was currently moving towards the gulch, apparently on joint set (3), but that it would be totally removed by the planned excavations. The excavation design is shown on Figure 3.

The investigation of the north side showed that there were three predominating joint sets (dip and dip direction (1) 50/130; (2) 70/260; and (3) 60/350). The combination of these joints and the poor condition of the rock at the upper levels where highly fractured dykes were identified, resulted in the slope design shown in Figure 4.

The closely jointed nature of the rock on both sides of the gulch necessitated strict specifications for excavation which included presplit blasting for all final faces, pre-support of bench crests with angled dowels, pattern dowel reinforcement for all excavated faces, shotcrete support where considered necessary on site, and careful setting and alignment of blast holes. The use of mesh and/or fibre-reinforced shotcrete was to be determined by the as-excavated conditions.

## EXCAVATION

The foundation excavations began on the south bank in late August, 1984 and on the north bank soon afterwards. The south excavation was completed on 22nd November, 1984 after the onset of winter conditions. The northern excavation was completed by the same date but further foundation treatment continued until mid-January, 1985. Additional recommended support measures were installed during the fall of 1985. During excavation, significant geological features were encountered which required design modifications, they vindicated the decision to have a geotechnical engineer on site throughout construction. The features included a much more extensive disturbed zone on the south bank than had been indicated by the mapping and drilling program, a deep glacial meltwater channel at the 1158 m level on the north abutment and the presence of open joints (relaxed ground) on the 1130 m bench, also on the north bank.

### South Bank

When the excavations had progressed down to the 1154 m level on the south bank, a major (1 m wide) tension crack and disturbed rock zone was exposed. The excavation of subsequent benches showed that sliding had occurred towards the gulch on a unique fault plane (45/038) and that the direction of sliding was towards the gulch and not towards the bridge excavation; there did not appear to be a mechanism which could result in hazard to the bridge foundation provided the fault plane was uniform in attitude and did not lie below the springing. The progressive tracking of the fault with each successive bench excavation showed that the springing foundations were in undisturbed ground (see Figure 3). The closeness of the jointing throughout the sequence both above and below the plane of movement, and the relative insignificance of the shearing and disturbance along the actual plane, did not permit the identification of this feature during the investigation. In outcrop the fault plane was entirely masked by colluvium. Some difficulty was experienced in excavating the broken ground. However, a stable design was achieved by excavating back to the tension crack at the crest of the slope and developing rubble slopes at 1:1.5 (33°) down to the toe of the planned slope on the 1130 m bench.

North Bank

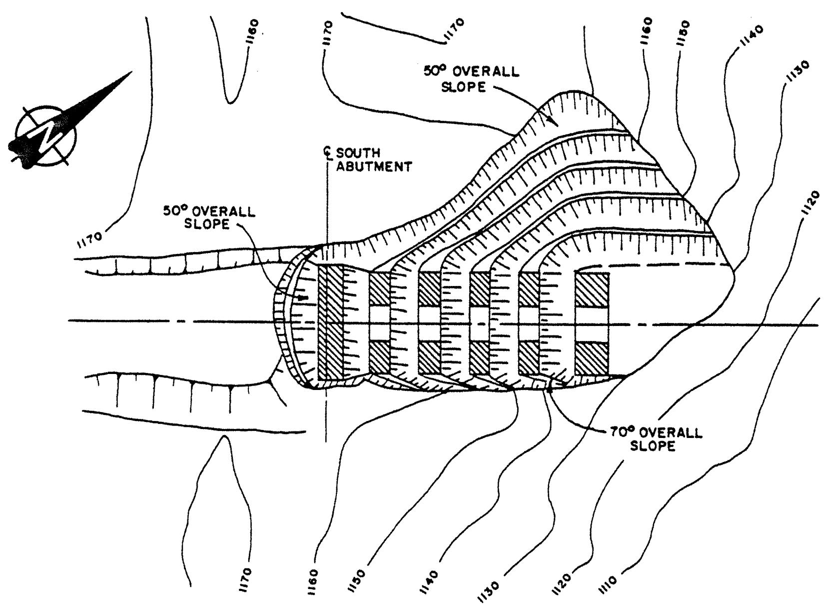
As the north bank was stripped of colluvium at the 1158 m level, a deep channel was exposed running parallel to the gulch; it was infilled with sands and gravels (see Figure 4). This glacial meltwater feature had been eroded parallel to one of the major joint sets and had exploited the broken ground produced by the diabase dykes. The channel was cleaned out and a double bench developed such that the piers planned to be founded at 1151.5 m elevation had to be lengthened and founded at 1145 m elevation. However the rock face between these benches was formed by the north side of the channel and was steeper than the planned design, it was even undercut in places. Consequently, extra reinforcement was required for that slope and for long-term stability it was recommended that a tied-back wall should be installed.

When the 1130 m bench was excavated for the arch springing the foundation was scaled back to sound unjointed rock both on the bench and bench face against which the concrete was to be poured. On the bench, but outside the springing, open joints were exposed; one joint traversed the edge of the foundation. An immediate program of further rock dowelling and grouting was undertaken and detailed geological mapping carried out on the slopes below the 1130 m level. In the subsequent field season shotcreting was carried out to prevent degradation of the slope in areas of fractured and faulted rock.

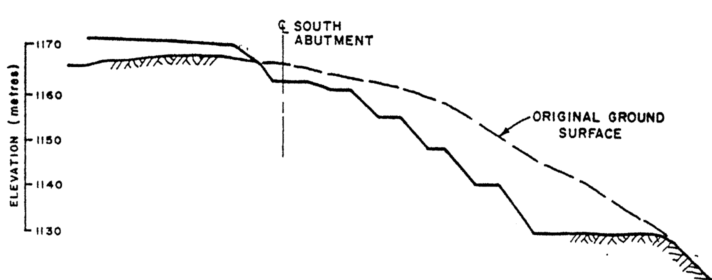
## SLOPE STABILIZATION

On both banks of Dry Gulch the rock is mostly closely (6-20 mm) to moderately closely (20-60 mm) jointed. There were few joint blocks which were in danger of imminent failure and were of a large enough size to require specific stabilization in the form of tensioned bolts. Overall support to the rock mass was provided by pattern dowels, augmented by extra dowels as necessary, and fibre-reinforced shotcrete. Dowels were 25 mm deformed Dywidag bars installed in fully grouted 38 mm holes. Resin grout cartridges were used in most early holes but were replaced by cement grout at the lower levels where longer bolts were required and the use of resin was not feasible with connectors. Installation was according to the manufacturer's specifications and proof tests indicated

EXCAVATION ON 1140m BENCH




PLANNED DESIGN




EXCAVATED DESIGN

Figure 3. Dry Gulch south springing.

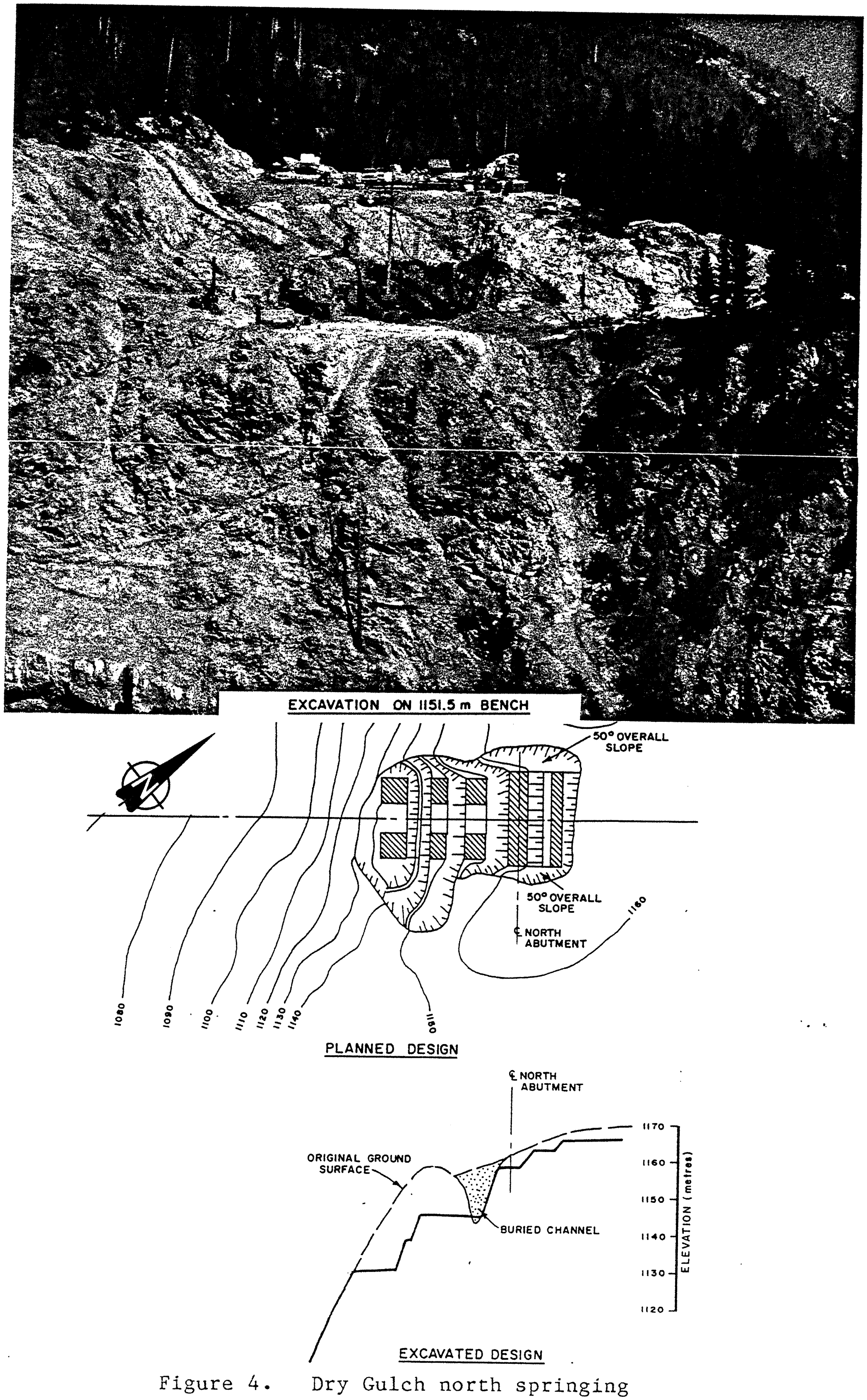



Figure 4. Dry Gulch north springing

that the dowels held a design load of 120 kN. Careful surveillance of these slopes will be required to check on potential corrosion of the bars and the need for future additional or replacement reinforcement.

Shotcrete reinforced by Dramix fibres was used for additional slope support until low temperatures precluded its use. Steel mesh was used for temporary protection in the more blocky ground where shotcrete was less appropriate and for temporary control of loose rock until shotcrete could be placed during the 1985 season.

The proposed tied-back wall on the 1145-1158 bench face was partially installed during 1985 and is to be completed during 1986.

No instrumentation has been installed in these slopes to the present time. However it is recommended that routine surveillance should be undertaken to check on the potential deterioration of the slopes with time.

## OTTOMITE BRIDGE

The Ottomite Bridge is a three-span, pre-cast concrete box girder bridge with a total length of 95 m and a width of 28.5 m. The bridge is on one of the steepest portions of the highway where the grade is 8 per cent as the highway climbs out of the Boston Bar Creek Valley to the Pass at elevation 1,000 m (Figure 1).

### DEBRIS FLOWS

The bridge crosses two stream channels which are separated by a narrow ridge of rock on which the two piers are located (Figure 5). It was determined in the initial route evaluation studies that there was a high potential for debris flows in the two streams. Experience on other highways in the Coastal Mountain ranges (Thurber 1985), as well as a number of other countries (Fryxell et al 1943, Gagoshidze * 1974, Mears 1977), has shown that debris flows can be extremely destructive. Consequently, measures have been taken to protect the Ottomite Bridge from such events.

Debris flows are a highly fluid mixture of water, solid particles and organic matter. For turbulent flow conditions the water content is as high as 70 per cent to 80 percent, and the solid material ranges from clay and silt size particles up to boulders several meters in diameter. The organic matter can include bark mulch as well as large trees and logs swept from the sides of the channel. Debris flows usually occur during periods of intense rainfall or rapid snow melt and a possible triggering event can be the failure of a temporary dam formed by a mud slide or log-jam, that releases a surge of water and solid material. Where such flows originate in streams with gradients steeper than about 20 to 30 degrees, they move at velocities at great as 10 to 20 m/sec and at this speed, material is scoured from the base and sides of the channel so that the volume of the flow increases as it descends. Debris

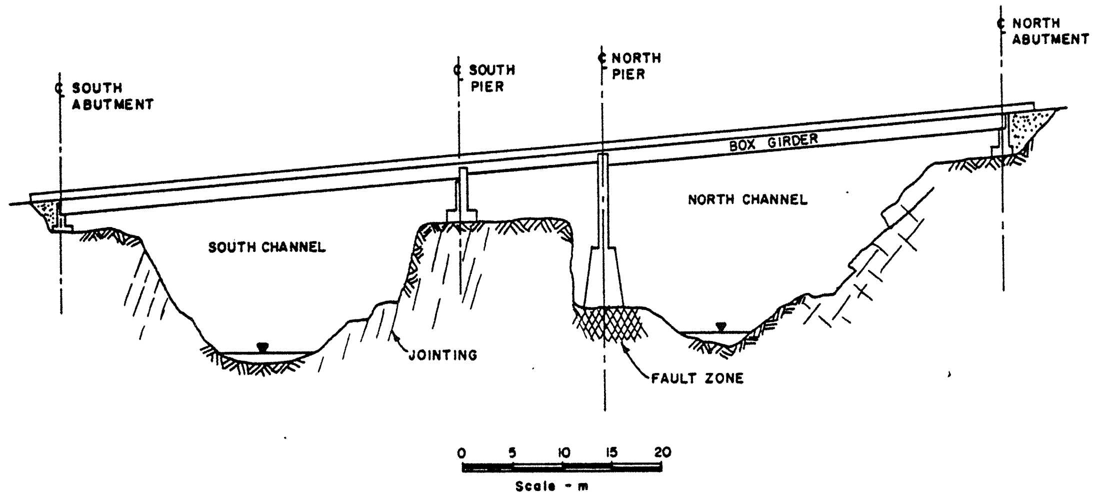



Figure 5. Ottomite Bridge

flows with volumes of as much as 20,000 cu m have been commonly recorded, and such events can be extremely dangerous, both in terms of flooding and destruction of man-made structures that lie in their path.

The Ottomite Bridge is located near the base of a mountain slope that rises to height of about 1,000 m above the bridge at an average slope of between 20 and 30 degrees. This is also a high precipitation area where the total snow accumulation can reach 5 m and heavy rain storms occur. All these conditions produce a high risk of occurrence of debris flows, and in fact, evidence of a recent, small flow was observed at the site in the year prior to construction.

The flow volumes used to design the channel dimensions were 15,000 cu m for the south channel and 8,000 cu m for the north channel, both at a flow velocity of 6 to 7 m/sec. In order to ensure that this volume would pass safely under the bridge, it was necessary to enlarge the existing channels and cut the channel bottoms at a uniform slope of 10 degrees so that there would be no build up of material at the bridge. The design channel section had a 6 m wide base, 45 degree side walls and the depth was determined by providing a minimum clearance of 7.7 m under the south span and a clearance of 7.2 m under the north span.

The widening and deepening of the channel was carried out by an extensive blasting operation that started at a distance of about 100 m above the bridge to produce the required uniform gradient. In addition, the slope below the bridge was opened out to provide a catchment area.

Another important aspect of the design was to locate the abutments and piers on rock close to road elevation and out of the channels so that there would be no danger of the footings being swept away in the event of a debris flow.

## FOUNDATION DESIGN

During the route evaluation studies, some mapping of outcrops in the vicinity of the bridge was carried out. Exploration was limited to this work because there was sufficient exposure to ascertain the general nature of the rock. Also, access for a drill rig was difficult, even for helicopters and clearing of vegetation.

The mapping showed that the rock was a coarse grained, massive granite which was very strong (compressive strength approximately 200 MPa) and generally unweathered. The rock strength far exceeded the foundation stresses, so there was little concern for the bearing capacity of the foundation rock. However, the rock contained several sets of joints faulting and some alteration; it was these features that governed stability. The joints formed three, approximately orthogonal sets of fractures which were both planar and continuous over lengths of several tens of meters. Sheeting joints parallel or sub-parallel to the ground surface were the most persistent structures. The combination of these joints formed slabs and large wedges with volumes as large as 500 cu m, the stability of which depended partly on their orientation with respect to the cut sides of the channels. Figure 5 shows the variation in geological conditions at each footing. This required that different stabilization measures be applied to suit each condition, as described below.

The excavation that had been carried out to deepen and widen the channel had increased the height of the cut faces on which the pier and abutments were founded. Also, the blasting had loosened and broken the rock to some degree so the new slopes were less stable than those in the original channel walls. It was apparent after the excavations had been completed that the bridge site was on the alignment of a zone of geological disturbance which has determined the alignment of Boston Bar Creek.

## SOUTH CHANNEL

In the south channel, one joint set (1) dipped at between 70 to 80 degrees to the south and was aligned approximately parallel to the channel. Thus, on the south side of the channel, these joints dipped into the slope and a steep stable face was formed which required no support or reinforcement.

On the north side of the channel, this joint set dipped out of the slope and the face was formed by these joints. At the eastern end of the pier the sheeting joints (2) dipped at about 30 to 35 degrees to the east and formed surfaces along which blocks of rock on which the east end of the pier was founded to slide (see Figure 6). The release surfaces on the sides of these blocks were set (1) aligned parallel to the channel and a fault (F1) with a near vertical dip striking at right angles to the channel.

Before construction of the foundation started, several loose slabs of rock formed by set (2), were scaled from the top surface of the footing

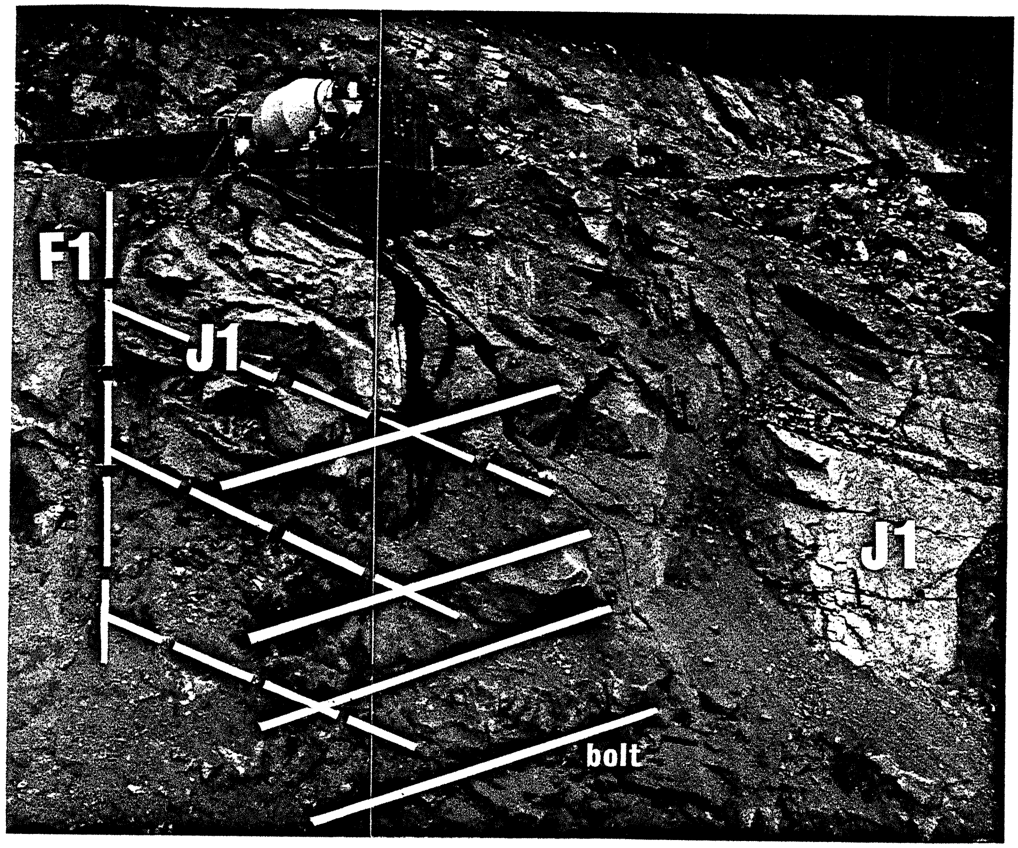



Figure 6. Ottomite Bridge south pier showing joint planes and rock bolt locations.

area, following which the bridge was constructed with no indication of movement of the foundation. Stability analyses of the blocks in the foundation were carried out using a friction angle of 41 to 44 degrees for the planar joint surfaces and a bridge loading which consisted of a vertical force of 8,300 kN due to the weight of the structure and a horizontal earthquake load of 320 kN. These calculations showed that the factor of safety for the loaded but unsupported foundation was about 1.2 to 1.3. This was not considered adequate for long term stability during which time the foundation would be subjected to traffic vibrations, ice and water forces and possibly earthquakes. It was decided to raise the factor of safety to about 1.7 by installing tensioned bolts through the blocks under the east end of the pier and securing them in stable rock near the base of the channel (Wyllie 1979).

Rock bolt installation is planned for 1986 before heavy traffic loads are imposed. A total of 13, 10 m long bolts each with a working load of 260 kN will be installed using a crane on the bridge to support equipment to drill holes in the steep rock face below the footing. Because of the long service life of these bolts, particular attention will be paid to the corrosion protection of the steel by grouting on a plastic sleeve before the bolt is installed.

NORTH CHANNEL

In the north channel, the major geological feature was a fault zone that intersected the eastern end of the north pier. This zone was markedly relaxed showing open joints and evidence of movement towards the creek. It was considered that this material was not sufficiently stable against sliding or erosion so it was removed down to a level close to the the channel bottom (Figure 7). The consequence of having this 12 m deep step in the foundation with a considerable greater mass of concrete at the downstream end of the pier, was that under earthquake loading there was a possibility of overturning movements. This situation was corrected by installing six, 10 m long bolts through the pier in a direction at right angles to the bridge alignment. During pouring of the concrete, sleeves were left through the structure so that holes for the bolts could be drilled later into the foundation rock. When the rock bolts had been installed through the footing and anchored in sound rock upstream of the fault, they were tensioned against the concrete face of the pier.

On the north side of the channel, the main set of joints dipped at an angle of about 45 degrees to the south and these formed the face of the cut slope. By locating the abutment at a distance of about 5 m behind the crest of the cut there was no risk of instability due to failure of slabs of rock on the face.

Stabilization measures that will be taken to ensure the long term stability of this face will be to scale loose rock from the face, install dowells through potential slabs and to shotcrete the face.

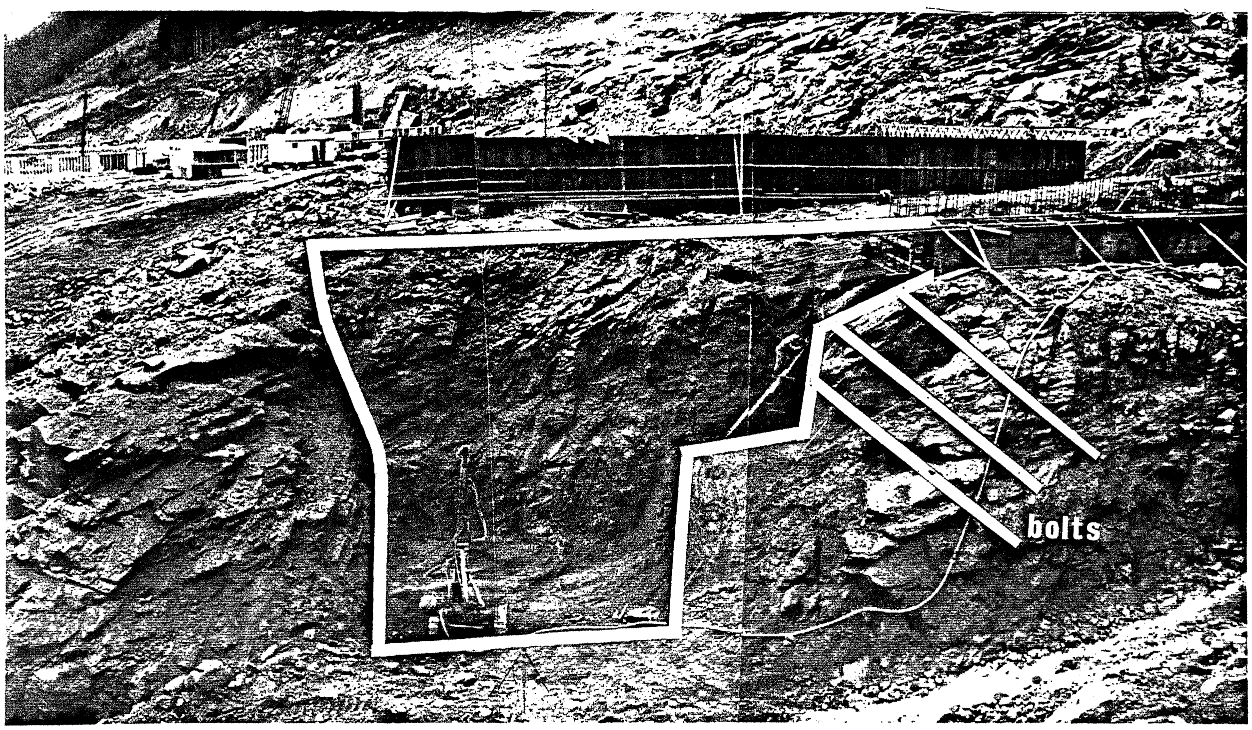



Figure 7. Ottomite Bridge north pier showing fault zone excavation and rock bolt locations.

## CONCLUSIONS

From the experience on the Coquihalla Highway, it is apparent that bridge foundations on rock in mountainous terrain must be treated with caution. Bedrock levels, rock quality and structural stability of the rock should be assessed with care especially where outcrop is poor. If the design investigations do not permit full and detailed assessment of the foundation conditions, a flexible approach to the design must be adopted wherever possible. The form of contract must permit design changes during construction. This approach is well accepted in many aspects of geotechnical engineering works (dam foundations, tunnels, etc.) but is less common or acceptable where pre-ordering of structural components is required. The close involvement of a geotechnical engineer during excavation is essential for assessment of in situ conditions and design of rock stabilization measures.

## ACKNOWLEDGEMENTS

We would like to express our appreciation to the British Columbia Ministry of Transportation and Highways, under the direction of Mr. T.R. Johnson, Deputy Minister, for their advice and assistance during the design and construction of the bridges.

Also, we acknowledge the two bridge designers for whom we acted as geotechnical engineers on the Dry Gulch Bridge, Reid Crowther and Partners Ltd. and on the Ottomite Bridge UMA Engineering Ltd. During construction, Golder Associates provided technical assistance directly to the B.C. Ministry of Transportation and Highways who supervised the implementation of the designs.

## REFERENCES

1) Field, J.A., Egan, T.A.P., Hallam, W.J. and Woodburn, D.L., 1978. "Hope-Merritt" Highway Corridor, Surficial Geology". Internal Report, Ministry of HIghways and Public Works, Geotechnical and Materials Branch

2) Fryxell, F.M., and Horberg, L (1943). "Alpine mudflows in Grand Teton Mational Park, Wyoming:. Bull. Geol. Soc. Am. (54), 457-472.

3) Gagoshidze, M.S., and Natishvili, O.G. (1974). "Nature of the Alma-Ata Mudflow". Translation in Soviet Hydrol. (5), 344-350.

4) Mears, A.I. (1977). Debris-flow Hazard Analysis and Mitigation: An Example from Glenwood Springs, Colorado". Col. Geol. Surv. Inf. Ser 8, 1-45.

5) Monger, J.W.H., 1969. "Hope Area; West Half", Geological Survey of Canada. Paper 69-4).

6) Thurber Consultants Ltd. Report to Ministry of Transportation an Highways on Debris Flows on the Coquihalla highway, March, 1985.

7) Wyllie, D.C., 1979. "Fractured Bridge Supports Stabilized under Traffic", Railroad Track and Structures, July 1979.